

POLAVARAM HEP (12 x 80 MW)

Design Basis Memorandum for Pressure Tunnel and Steel Liner

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1 INTRODUCTION

Andhra Pradesh Power Generation Corporation Limited (APGENCO) has awarded the EPC Works for Construction of Polavaram HEP (960 MW) to M/s Navayuga Engineering Company Ltd. (NECL). M/s NECL has retained M/s SMEC India Pvt Ltd (SMEC) as Detailed Design Consultant for the Engineering of the Project.

The Polavaram Project is a multipurpose project, proposed across Godavari River near Polavaram village, situated at about 42 km upstream of Godavari Barrage at Dowlaiswaram. The Longitude and Latitude of the Project are 81° 46' East and 7° 13' N respectively.

The FRL and MDDL of the Polavaram Dam are El 45.72m and El 41.72m respectively. The Project Components for Power development are proposed across the existing spur on left bank. With normal TWL at El 18.20m and utilizing the gross head of 27.52m, the total installed capacity is proposed to be 960 MW having 12 Units of 80 MW each. The design discharge is 331 cumec per unit for 12 Units.

The works of the Dam and Spillway Complex had been taken up by the Irrigation Department under a separate Contract and excavation is in progress. The proposed Powerhouse Complex is aligned on the left bank of the river Godavari and basic excavations for the Approach Channel / Intake Pool, Intake Structure Slope, Power House Complex including Service Bay / Unloading Bay, Tail Race Channel etc are already in progress along with the Dam works. The construction of Embankment along the Tail Race Channel is also under execution.

The main components of the Powerhouse Complex are as follows:

1. Concrete lining of the Approach Channel / Intake Pool
2. Power Intake Structure
3. 12 Nos. Pressure Tunnels of 9m diameter (finished) each with Steel Liners
4. Power House (Unloading Bay / Service Bay / Unit Bay / Auxiliary Rooms including space for Transformers and GIS) to accommodate twelve generating units of 80 MW each
5. Lining of Tail Pool and Tail Race Channel

2 OBJECTIVE OF THE REPORT

The present Report outlines the Hydraulic & Structural Design Basis, Philosophy, Data, Loads, Load Combinations etc. to be adopted during the Detailed Design and preparation of Construction Drawings for different components of the Project. This report would deliberate the understanding of the particular component and expected loading conditions during the life cycle of the Project. The Report would also avoid any basic philosophical changes at later stage of detailing which normally lead to reworking and project delays.

This DBM covers the design criteria of rock support & steel liner for Pressure Tunnel and concerns the following:

- a) Excavation and Rock Support Design

- b) Steel Liner Design

3 BRIEF OUTLINE OF THE PROJECT AREA COVERS IN DBM

For Polavaram Hydroelectric Project, 12 numbers of Pressure Tunnels of 9.0m finished diameter has been proposed to draw the design discharge of 331.0 cumec for each unit with installed capacity of 80 MW. Excavation profile of each tunnel is proposed to be circular in shape with 300mm thick backfill concrete around the steel liner. Each pressure tunnel takes off from the Intake at RD 316.4m with center line elevation at El 26.30m leading to Concrete Spiral Casing transition at RD. 444.0m with center line of machine at El.10.80m. The tunnel alignment has two bends of 8.45 degrees. The pressure tunnel is proposed to be steel lined all along its length. The layout & longitudinal section of pressure tunnel is attached at Annexure-2.

4 PROJECT REFERENCE DOCUMENTS

The below documents have been made available to the Designer for corroboration of this Design Basis Memorandum and further detailing:

- EPC Bid Documents including Drawings, Technical Specifications and subsequent Amendments / Clarifications / Additional Documents (if any) issued during Bidding Process
- Drawings as submitted with the Tender
- Design Approach and Methodology as prepared during Tender Stage

5 STANDARDS, MANUALS & OTHER REFERENCES

Refer **Annexure-1** for list of Manuals and various Codes to be consulted and referred during the detailed design. Apart from these listed codes and other international standards, the relevant engineering literatures, manuals and other standards/codes may be referred as per works requirement.

Annexure-2 encloses Pressure Tunnel Layout Plan as 7061507-C351 and Pressure Tunnel Section as 7061507-C352.

6 UNIT SYSTEM

For all technical documents, the SI system will be used. The units of some important parameters are the following:

Mass	Kg or MT
Loads / Forces / Shear	kN, kN/m, kN/m ²
Bending Moments	kN-m
Pressures, Stresses	N/mm ² (= MN/m ² = MPa) or kN/m ² (= kPa)
Area	m ² or sqm
Volume	m ³ or cum
Unit Weight	kN/m ³
Discharge (Rate of flow)	m ³ /s or cumec

Velocity	m/s
Temperature	°C
Angle	Degrees (°)

7 DESIGN DATA

The following design data shall be used in the analysis:

- Full Reservoir Level (FRL) : El 45.72 m
- Minimum Drawdown Level (MDDL) : El 41.15 m
- Design Discharge for 12 units : 3972 Cumec
- Net head corresponding to one unit running condition : 30.70 m

7.1 SEISMIC ANALYSIS

The project site is located in seismic zone III as defined in IS 1893. The seismic horizontal coefficient proposed to be adopted shall be as per IS 1893 (Part IV):2005

$$A_h = \frac{Z}{2} \times \frac{S_a}{g} \times \frac{I}{R}$$

Where,

Z = Zone factor as per Annex A of IS 1893 (Part-IV):2005

I = Importance factor for category 2 given in Table 2 of IS 1893 (Part-IV):2005

R = Response reduction factor given in Table 3 of IS 1893 (Part-IV):2005

Sa/g = Average response acceleration coefficient given in given in Annex B of IS 1893

(Part-IV):2005

The value of seismic horizontal coefficient is

$$A_h = \frac{0.16}{2} \times 2.5 \times \frac{1.75}{3.0}$$

$$= 0.117$$

Seismic Coefficients

Event	DBE
Design Horizontal acceleration	0.117
Design Vertical acceleration (2/3 rd of Horizontal)	0.078

8 MATERIAL PROPERTIES

The following sections generally define the main construction material properties to be used for the design. These conform to the specifications for materials included in the technical specifications.

CONCRETE

The grade of concrete shall be according to IS 456:2000. The M20 concrete has been envisaged for backfilling i.e. the concrete has characteristic compressive strength of 20 MPa for the 150 mm cube at 28 days. Unit weight of concrete has been adopted as 24 kN/m³.

The modulus of elasticity of concrete shall be $5000 \sqrt{f_{ck}}$ MPa as per section 6.2.3.1 of IS: 456 – 2000, where f_{ck} is the 28 days characteristic strength. Poisson's ratio of concrete will be taken as 0.25. The concrete shall be considered as elastic and isotropic material with homogeneous behavior.

The value given above shall apply for normal operation conditions for permanent structural loads. The compressive and tensile strength of the concrete will conform to relevant Indian Standard.

REINFORCEMENT

Steel for reinforcement will be of grade Fe 500 (with yield stress of 500 MPa) cold drawn, high strength, deformed bars in accordance with IS: 1786-2008 in case it is required to be used at any location. Although, presently it is not envisaged along the Pressure Tunnel.

The minimum reinforcement to be provided in each structural component shall be as defined in IS: 456 – 2000.

STEEL LINER

As given in technical specification, ordinary grade steel of IS 2062, Grade B shall be used for the pressure tunnel steel liner. Yield strength and ultimate tensile strength of the steel are discussed in para 10.5.1.

ALLOWABLE STRESSES

Allowable stresses in concrete and reinforcement steel shall be as per IS 456:2000.

UNIT WEIGHTS

The following unit weights shall be used for design of structure:

- Unit weight of mass concrete : 24.0 kN/m³
- Unit weight of reinforced concrete : 25.0 kN/m³
- Unit weight of water : 9.81 kN/m³
- Unit weight of structural steel : 78.50 kN/m³
- Unit weight of saturated soil : 22.0 kN/m³
- Unit weight of rock : 26.0 kN/m³

9 DESIGN METHODOLOGY FOR TUNNEL ROCK SUPPORT

DESIGN PHILOSOPHY

To design any underground structure, it is important to ensure two types of stability:

- Local Stability:** While an underground opening is being excavated, the excavation surface can intersect different sets of rock joints and lead to formation of wedges on the roof or on the side walls of the opening. To check the stability of such wedges, UNWEDGE software from Roc Science shall be used. UNWEDGE is a program used to analyse the geometry and the stability of rock wedges, defined by intersecting structural discontinuities in the rock mass surrounding an underground excavation.
- Global Stability:** Global stability of rock mass surrounding the excavated opening is defined in terms of stresses and deformation around the opening. To assess the global stability of the opening a stress-deformation analysis shall be carried out using the 2D-FEM program PHASE2 from Rocscience.

The adopted design philosophy is summarized in figure below:

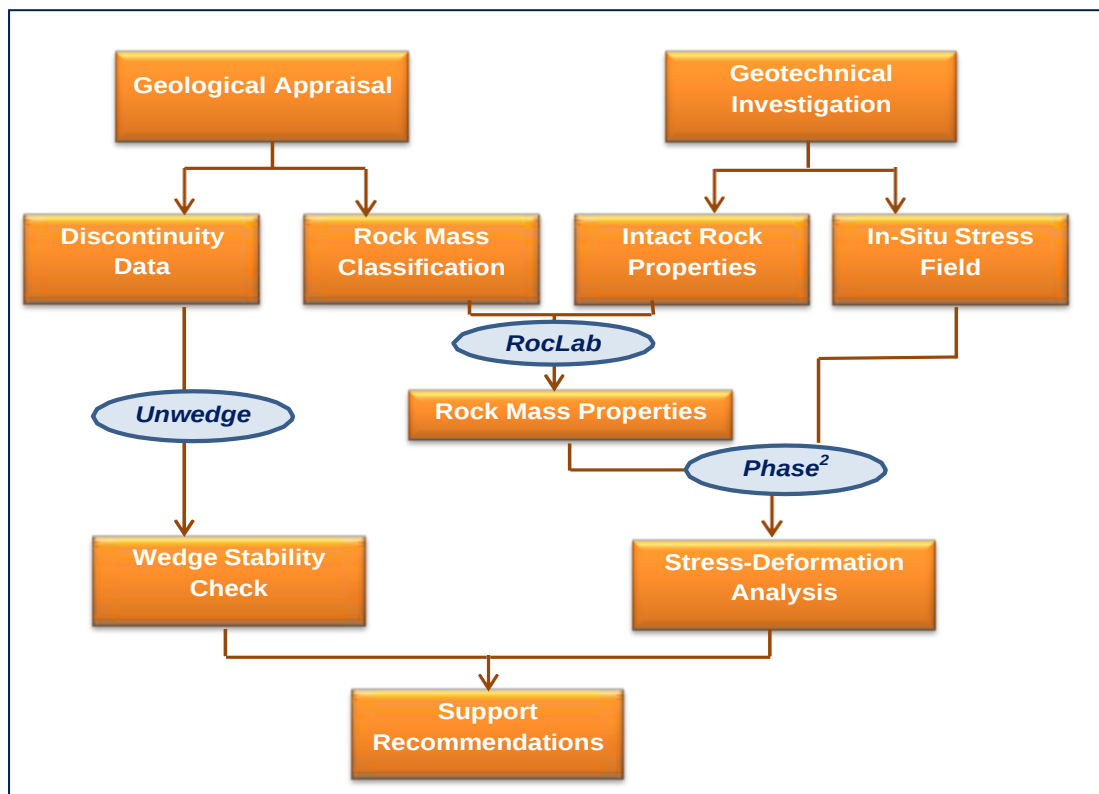


Figure 1 Design Process Chart

GEOLOGICAL APPRAISAL

The pressure tunnels are expected to be aligned through the fresh bedrock of Gneisses except tunnel No. 1 and 2 (near to dam side) towards the southern end of Intake and Powerhouse where the initial stretch of these tunnels on both the sides of the spur are expected to be located in the weathered bedrock. The vertical ground cover over the tunnels is increasing from tunnel no. 1 to 12. The cover is expected to be low on tunnel 1

and 2 while likely to be adequate on remaining tunnels. The lateral rock cover between the tunnels is also limited due to machine centre line spacing at 24m and tunnel excavated size is around 9.6m.

GEO-MECHANICAL PROPERTIES

Detailed geotechnical characteristics would be required for refinement of the geotechnical model during the various detailed design and construction phases as part of the design development in order to take full account of interactive and progressive effects leading to strength and/or stiffness degradation.

IN-SITU STRESS

For analysis, gravity field stress option shall be used to define the in-situ stress field.

WEDGE STABILITY ANALYSIS

The analysis of the geometry and the stability of underground wedges defined by intersecting structural discontinuities in the rock mass surrounding the pressure tunnel shall be performed by UNWEDGE software developed by Roc science.

Following assumptions shall be considered:

- i. All the discontinuity surfaces are perfectly Planar
- ii. Wedges are subjected to only gravitational loading. Consideration of in-situ stress helps the wedges to stabilize but will be neglected for long term stability

STRESS DEFORMATION ANALYSIS

For the stress-deformation analysis, two-dimensional numerical modelling shall be done using Phase2 program from Roc science.

ASSUMPTIONS IN PHASE2 ANALYSIS

- The rock mass surrounding the excavation assumed to be homogenous and isotropic in all directions.
- Rock mass shall be modeled as a continuum following the Mohr-Coulomb failure criterion.
- Blast damage or Disturbance zone shall be considered in the first 2m of the rock surrounding the excavation.
- Support installation is assumed to be implemented after excavation of one round which results in a relaxation of stresses from 70% to 90%.

ROCK SUPPORT

Based on the stress deformation analysis in Phase2, a range of rock support measures shall be considered for the different classes of rock. These include:

- Shotcrete / SFRS
- Systematic rock bolting
- Rock anchors

- Lattice Girder / Steel Ribs

Other, more sophisticated, rock support measures are available in the addition to the above. However, such measures will be presented and specified in detail design documents related to special cases, once they occur.

10 DESIGN METHODOLOGY FOR STEEL LINER

DATA

Design Discharge for 12 units	: 3972 Cumec
No. of machines	12
Center line of machines	: 10.80 m
Net head corresponding to one unit running condition	: 30.70 m
Manning's coefficient	: 0.011
Diameter of Pressure Tunnel Steel Liner	: 9.0 m
Length of the Steel Liner	: 127.607m
Bulk Modulus of water (E_w)	: 2×10^9 N/m ²
Modulus of Elasticity of steel (E_{st})	: 2×10^{11} N/m ²

DESIGN LOADS

The following loads shall be considered in the design:

INTERNAL WATER PRESSURE

The steel liner will be subject to internal water pressure corresponding to the maximum water level in reservoir while system is operating (i.e. EL 45.72m) and increase in pressure due to water hammer.

EXTERNAL PRESSURE

Various types of external pressure of different magnitudes act on the ferrules under different working conditions. They are enumerated in following paragraphs:

EXTERNAL PRESSURE DUE TO WATER

The steel liner of the pressure tunnel will be subject to unbalanced external pressure from the surrounding ground water when pressure tunnel is dewatered. As per IS 11639 (Part 2), the steel liner should be designed for the external pressure which is either the difference between the ground level vertically above the pressure tunnel and the level of section being designed or the maximum level from which the water is likely to find its way around steel liner, whichever is less. For Polavaram HEP, at start of pressure tunnel (i.e. u/s face) maximum water level is at FRL EL.45.72m whereas on d/s face (i.e. end of the pressure tunnel) pressure release line is available at EL.20.0m all along the length of power house. Therefore, the external water pressure follows the gradient line joining between the EL.45.72m at start to the EL.20.0m at end.

EXTERNAL PRESSURE DUE TO CONTACT GROUTING

As per the Technical Specifications, contact grouting & *consolidation grouting* is envisaged around pressure tunnel. The minimum external pressure around pressure tunnel is about 20.48m which is equal to a pressure of 204.8 kPa. The usual grouting pressure used for contact grouting is about 200 kPa which is less than this pressure head. As such, the grouting pressure does not govern the design of steel liner.

EXTERNAL PRESSURE DUE TO WET BACKFILL CONCRETE

It is envisaged that the annular space between the liner and the excavated surface of the pressure tunnel will be backfilled with M20 concrete. The green concrete will exert hydrostatic pressure during the interval between pouring and setting. As the minimum external water pressure head around steel liner is about 20.48m which is much higher than this. Hence, this case does not govern the design.

10.2.3 ROCK LOADS

Sufficiently long time is likely to lapse between the excavation of the pressure tunnel and erection of the liner. During this period, the excavated surfaces would undergo deformation, if any, and would get stabilized. As such, no rock load is expected to come on the liner after its installation and hence, need not be considered.

LOADING CONDITIONS

The following load conditions, as recommended in IS 11639 (Part 2): 1995, shall be considered:

NORMAL LOAD CONDITION

The normal case includes maximum internal pressure along with increase due to water hammer pressure rise. As per IS 11639 (Part 2): 1995, maximum water level at the reservoir should be joined with the maximum pressure level due to water hammer at the power house and assumed to vary linearly along the length of the pressure tunnel, the difference between this line and the level of the design section of pressure tunnel is to be considered as design internal pressure.

INTERMITTENT CONDITION

Intermittent condition includes the filling & draining of pressure tunnel. During filling, the internal pressure exerted on the steel liner shall be less as compared to the pressure exerted during normal condition as explained above. However, during draining, the pressure tunnel may be subjected to external water pressure and the same needs to be checked for buckling. Hence, the intermittent loading condition has been considered for the design.

EMERGENCY CONDITION

In case of emergency condition, one gate is assumed to be closed in critical time of penstock ($= 2L/a$, where 'L' is length of the pressure tunnel and 'a' is the pressure wave velocity). The critical time for the present case comes out to be about 1.08 seconds, which is very much less than the minimum time of closure equivalent to 5.27 secs. The turbine governor is unable to close the wicket gates within this short time of 1.08 seconds. As such, emergency condition is not applicable and shall not be used for the design of steel liner.

EXCEPTIONAL CONDITION

This case refers to malfunctioning of control equipment in the most adverse manner, resulting in odd situation of extreme loading. Since these conditions of operation require a complete malfunctioning of governor control apparatus and other safety devices at the most unfavorable moment, the probability of obtaining this type of operation is extremely remote. The IS code clearly states that this case is not to be considered for design of steel liner.

WATER HAMMER

NORMAL OPERATING CONDITION

For Normal operating condition, the increase in pressure is given by the equation

$$= \frac{2 \cdot V \cdot L}{g \cdot T_c}$$

Where V = Velocity in pressure tunnel

$$= \frac{331}{(\pi \cdot 9^2 / 4)} = 331 / 63.617 = 5.20 \text{ m/sec}$$

L = Length of pressure tunnel

$$= (474.5 - 307.4) = 167.1 \text{ m}$$

T_c = Time of closure

For the purpose of water hammer calculation, it has been taken as 10 seconds which will be confirmed with E & M vendor.

$$\text{Increase in pressure head} = \frac{2 \cdot 5.20 \cdot 167.1}{9.81 \cdot 10} = 17.71 \text{ m}$$

$$= 17.71 \cdot 100 / 34.92 = 50.72 \%$$

The increase in pressure due to water hammer has been taken as 51 % of the net head at the machine centre line elevation. Moreover IS 12837:1989 also recommends a pressure rise of 30 to 50% for Kaplan turbines. Hence an increase of 51 % has been considered for the present analysis. The net head in normal loading will be $1.51 \times 34.92 = 52.73 \text{ m}$

MATERIAL AND DESIGN STRESSES

MATERIAL

As given in technical specification, ordinary grade steel of IS 2062, Grade B tested quality shall be used for the pressure tunnel steel liner. Yield strength and ultimate tensile strength of the steel are given in Table-1:

Table-1

Material	Yield Strength, f_y (MPa)	Ultimate Tensile Strength, f_u (MPa)
E 250 Grade B (IS: 2062 - 2011)	240.0 (for thickness 20 - 40mm)	410.0

DESIGN STRESSES

As per IS:11639 (Part 2), allowable stresses for embedded tunnels/penstocks shall be taken as:

- In normal operating condition, the design stresses should not exceed 1/3 of the minimum ultimate tensile strength or 60% of minimum yield strength, whichever is less.
- In intermittent condition, the design stresses should not exceed 40% of the minimum ultimate tensile strength or 2/3 of minimum yield strength, whichever is less.

Using the above criteria, in the present case, the following allowable stresses are obtained for design (shown in Table-2).

Table-2

Grade of Steel	ERW, E 300 Grade B (IS: 2062 - 2011)		
Yield Strength, f_y (MPa)	290		
Ultimate Tensile Strength, f_u (MPa)	440		
Operating Condition	Lesser of		Allowable Stress
Normal operating condition – Lesser of 0.6 f_y or $f_u/3$	174	146.67	146.67
Intermittent condition – Lesser of 0.67 f_y or 0.4 f_u	193	176	176

DESIGN CRITERIA

PLATE THICKNESS

Commonly available plate thicknesses have been identified for the pressure tunnel. The maximum thickness of plate to be used for fabrication of penstocks shall be limited to 36 mm, based on practical considerations of plate bending and welding, as plates beyond this thickness would require stress relieving.

FERRULE WIDTH

Standard shell width shall be taken as 2500 mm. Two shells shall be joined in the shop to form a standard ferrule of 5000 mm length and transported to site for erection.

CORROSION ALLOWANCE

It is proposed that the inside surface of the penstock is painted with anticorrosive epoxy paint. Hence corrosion allowance is not considered as per cl 7.1b of IS 11639 (part-2).

INITIAL GAP BETWEEN LINER AND CONCRETE

The radial temperature gap, between steel liner and concrete, results from shrinkage of concrete during hydration. Radial gap could also be due to out of roundness of ferrule, thermal shrinkage/ expansion after installation, temperature difference during construction and operation of the power plant. The gap is formed mainly due to different co-efficients of thermal expansion of concrete and steel liner. This gap can realistically vary from 0 to 0.03% of radius of steel liner. In the present design, the initial gap has been assumed as 0.03% of radius of steel liner or in other words 0.3 mm per m of radius of liner. Contact grouting is envisaged around the steel liner to fill the gap between ferrule and back fill concrete and develop a close contact between liner and filler concrete.

MINIMUM HANDLING THICKNESS

Regardless of loading conditions, a minimum thickness of plate is required to provide rigidity for handling during fabrication and transportation. This thickness is directly related to the radius of ferrule to be fabricated. The minimum handling thickness “ t_0 ” is given by following formula

$$t_0 = \frac{R + 0.25}{200}$$

Where, R is the radius of penstock/ pressure tunnel in meters.

In the present case, the minimum handling thickness for pressure tunnel of 9.0 m diameter works out to be 23.75 mm, say 24mm.

JOINT EFFICIENCY

Pressure tunnel steel liners are fabricated with a combination of longitudinal and circumferential weld joints. Longitudinal joint of each shell is shop welded. Two shells shall be further joined by a circumferential weld in shop, forming a ferrule for

transportation to site. Each ferrule shall be erected at site and joined by a circumferential weld. All longitudinal and circumferential welds will be full penetration butt joints. All welded joints in shop will be checked by 100% radiographic test and 100% hydro static pressure test. All field joints shall be 100 % tested by ultrasound. For fully radio graphed weld joints, IS 2825 – 1969 (Table-1-1) recommends a joint efficiency of 100%. However, a joint efficiency of 90% has been adopted for the present case.

DESIGN FOR INTERNAL PRESSURE

Using the principle of Hoop stress, the plate thickness, t (mm), required to resist internal pressure, p (MPa), acting inside a circular steel tunnel of diameter, D (mm), is given by

$$t = \frac{p \cdot D}{2 \cdot \sigma \cdot \eta}$$

Where,

σ = Allowable stress (MPa) in steel.

η = weld Joint efficiency (90%)

Re-arranging the terms, we get the permissible internal pressure in meters of water head, H for a given plate thickness, t (mm), as

$$H = 2000 \sigma t / (9.81D)$$

Where t and D are expressed in mm and σ is in MPa.

DESIGN FOR EXTERNAL PRESSURE

For Polavaram HEP, natural ground level is above the external water pressure gradient line measured by joining the line between the u/s reservoir level (EL.45.72m) & d/s pressure releasing level (EL.20.0m) at generator floor . Hence, the external pressure at any location along pressure tunnel would correspond to the water pressure gradient line.

Adequacy of liner thickness under design external pressure can be ensured either with un stiffened liner or stiffened liner. It is understood that for unstiffened case results into thicker liner for resisting same external pressure than stiffened case, as stiffeners impart additional stiffness to ferrules.

- **Un stiffened liner:**

Failure of steel liner due to external water pressure occurs by buckling which forms mostly a single lobe parallel to the axis of the tunnel. Analytical methods have been developed by Amstutz, Jacobson and Vaughan for determination of critical buckling pressures for cylindrical steel liners without stiffeners. Amstutz method gives the minimum critical buckling pressure among all three methods. IS 11639 (Part 2) also recommends Amstutz and Vaughan methods; hence the buckling resistance of the pressure tunnel liner shall be calculated by Amstutz's method. After adopting a factor of safety of 1.5 over minimum value of critical external pressure the design value is adopted. The resulting value is then compared with actual external pressure acting on that particular reach of liner. If actual external pressure acting on liner is less than the above value, the un-stiffened liner is considered safe against external pressure,

otherwise thickness of un-stiffened liner is increased or stiffeners are provided or a combination of both is adopted for making it safe against external pressure.

ANNEXURE – 1

CODAL REFERENCES

REFERENCES FOR ROCK SUPPORT DESIGN OF PRESSURE TUNNEL

For the preparation of this document the following data sources were used:

- [1] Hoek, E., Carranza-Torres, C.T., and Corkum, B. (2002), Hoek-Brown failure criterion – 2002 edition. *Proc. North American Rock Mechanics Society meeting in Toronto in July 2002*. (Formulations on which RocLab was based).
- [2] Phase2 software Manual Ver. 8.00.
- [3] Un Wedge Manual Ver. 3.023
- [4] Roc Data Manual

REFERENCES FOR DESIGN OF STEEL LINER

- 1. Indian Standard IS 11639 -1995, “Structural design of Penstocks – Criteria, Part 2 (Burried/embedded penstocks in rock)”
- 2. Mosonyi, E. “Water Power Development”, Vol-2/A High Head Power Plants Akademiai Kiado, Budapest
- 3. Indian Standard IS 12837:1989, “Hydraulic Turbines for Medium & Large Power Houses- Guidelines for Selection”
- 4. Tunnels and Tunnels in Rock – EM 1110-2-2901-USACE

ANNEXURE – 2

DRAWINGS